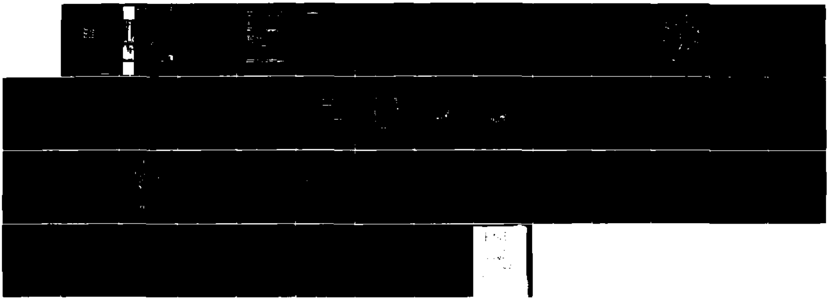
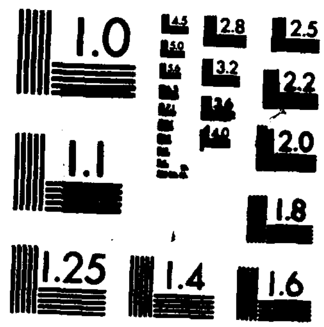
AD-A182 458 GEOTECHNICAL FEASIBILITY STUDY REPLACEMENT OR EXTENSION 1/1
OF THE CRANEY ISL..(U) ARMY ENGINEER WATERWAYS
EXPERIMENT STATION VICKSBURG MS GEOTE..
UNCLASSIFIED S J SPIGOLON ET AL. JUN 87 WES/MP/GL-87-9 F/G 13/2 NL

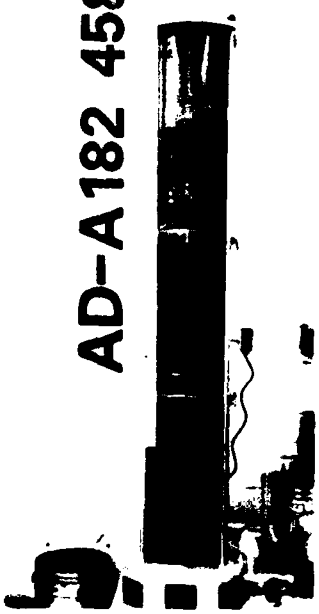




MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

2

US Army Corps
of Engineers



MISCELLANEOUS PAPER GL-87-9

GEOTECHNICAL FEASIBILITY STUDY REPLACEMENT OR EXTENSION OF THE CRANEY ISLAND DISPOSAL AREA NORFOLK, VIRGINIA

by

S. J. Spigolon, Jack Fowler

Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

June 1987

Final Report

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

and

US Army Engineer District, Norfolk
Norfolk, Virginia 23510-1096

GEOTECHNICAL LABORATORY

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

ADA182458

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1a REPORT SECURITY CLASSIFICATION: Unclassified

1b RESTRICTIVE MARKINGS: NA

2a SECURITY CLASSIFICATION AUTHORITY:

2b DECLASSIFICATION / DOWNGRADING SCHEDULE:

3 DISTRIBUTION / AVAILABILITY OF REPORT: Approved for public release; distribution unlimited

4 PERFORMING ORGANIZATION REPORT NUMBER(S): Miscellaneous Paper GL-87-9

5 MONITORING ORGANIZATION REPORT NUMBER(S):

6a NAME OF PERFORMING ORGANIZATION: USAEWES Geotechnical Laboratory

6b OFFICE SYMBOL (If applicable): WESGE-ET

7a NAME OF MONITORING ORGANIZATION:

6c ADDRESS (City, State, and ZIP Code): PO Box 631, Vicksburg, MS 39180-0631

7b ADDRESS (City, State, and ZIP Code):

8a NAME OF FUNDING / SPONSORING ORGANIZATION: US Army Engineer District, Norfolk

8b OFFICE SYMBOL (If applicable): NAO

9 PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER: IAO No. AD-86-3011

8c ADDRESS (City, State, and ZIP Code): 803 Front Street, Norfolk, VA 23510

10 SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO	PROJECT NO	TASK NO	WORK UNIT ACCESSION NO

11 TITLE (Include Security Classification): Geotechnical Feasibility Study Replacement or Extension of the Craney Island Disposal Area, Norfolk, Virginia

12 PERSONAL AUTHOR(S): Spigolon, S. J. and Fowler, J.

13a TYPE OF REPORT: Final report

13b TIME COVERED: FROM Jan 86 TO Oct 86

14 DATE OF REPORT (Year, Month, Day): June 1987

15 PAGE COUNT: 46

16 SUPPLEMENTARY NOTATION: Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161

17 COSATI CODES

FIELD	GROUP	SUB-GROUP

18 SUBJECT TERMS (Continue on reverse if necessary and identify by block number): Dikes; Dike stability; Dredged material; Dredging; Hydraulic fill

19 ABSTRACT (Continue on reverse if necessary and identify by block number)

Several alternatives were investigated to replace or extend the 2,500-acre Craney Island Disposal Area for continued dredged material disposal for the Norfolk harbor and channels. The disposal area was completed in 1957 and has been used continuously since that time and the dikes have been raised several times to the present el +26 ft Mean Low Water (MLW). Because of new plans to deepen the existing channel and continued maintenance dredging, the estimated storage life has greatly reduced.

Five alternate configurations of the extension of Craney Island to the west and/or north were investigated as to their estimated storage capacity, their effect on the hydrodynamics of the harbor and river system, and the stability and constructibility of the retaining dikes which is the subject of this report. It was determined during this investigation that it is technically feasible to design and construct the required dikes to

1)

(continued)

20 DISTRIBUTION / AVAILABILITY OF ABSTRACT: ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT ☐ DTIC USERS

21 ABSTRACT SECURITY CLASSIFICATION: Unclassified

22a NAME OF RESPONSIBLE INDIVIDUAL: Jack Fowler

22b TELEPHONE (Include Area Code): 601-634-2701

22c OFFICE SYMBOL: WESGE-ET

DD Form 1473, JUN 86 — Previous editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE

Unclassified

1

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

19. Abstract (continued).

el +8 ft MLW by hydraulic placement of sand; and 2) ~~It was also determined that~~ these dikes could be protected from erosion by the placement of riprap on the inside and outside slopes. The most economically feasible alternatives for storage capacity were configurations 5 and 2 which had an estimated cost per cubic yard of storage of $0.58 and $0.70, respectively. Keywords: Dike Stability.

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

PREFACE

This report presents the results of preliminary geotechnical engineering studies and gives estimated costs for embankment construction for five alternate disposal areas for the extension and/or replacement of the Craney Island Disposal Area located in Norfolk, Virginia.

This project was conducted for the US Army Engineer District, Norfolk (NAO), Norfolk, Virginia, and the US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, during the period January 1986 to October 1986. WES' work was authorized by IAO No. AD-86-3011 dated 10 December 1985.

Concept formulation and general supervision of the study were carried out by Mr. T. J. Szelest, NAO, and Dr. Jack Fowler, WES, under the guidance of Mr. Ron G. Vann, Chief, Dredging Management Branch, and Mr. Jack G. Starr, Chief, Engineering Division, NAO. Onsite technical investigations were conducted by Mr. M. T. Byrne and by Mr. D. A. Pezza, Chief, Geotechnical Branch, NAO.

This report was written by Dr. S. J. Spigolon and Dr. Jack Fowler under the general supervision of Mr. G. B. Mitchell, Chief, Engineering Group, Soil Mechanics Division (SMD), Mr. C. L. McAnear, Chief, SMD, and Dr. W. F. Marcuson III, Chief, GL.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

This report should be cited as follows:

> Spigolon, S. J. and Fowler, J. 1987. "Geotechnical Feasibility Study, Replacement or Extension of the Craney Island Disposal Area, Norfolk, Virginia." Miscellaneous Paper GL-87-9, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

GEOTECHNICAL FEASIBILITY STUDY
REPLACEMENT OR EXTENSION OF THE CRANEY
ISLAND DISPOSAL AREA, NORFOLK, VIRGINIA

PART I: INTRODUCTION

Background

1. The Craney Island Disposal Area is a 2,500-acre confined dredged fill disposal facility operated by the U.S. Army Engineer District, Norfolk (NAO). The facility is located adjacent to and north of Portsmouth, near Norfolk, Virginia, shown in Figure 1.

2. Craney Island is the major maintenance dredged material disposal area for the Norfolk harbor and channels. It was completed in 1957 and has been used almost continuously since then. The level of the perimeter dikes has been raised several times, reaching the present el +26 ft MLW. The original design was for an estimated 96 million cu yd capacity and a 20-year life for the facility.

3. Due to new plans for greatly increased harbor dredging, including both maintenance and channel deepening, the estimated life of the existing facility has been greatly reduced. Studies of the disposal problem (Norfolk District, 1985) have recommended extension of the Craney Island Disposal Area to the west and/or north. Proposals for extension have resulted in five alternative configurations, shown in Figure 2, having different estimated storage volumes.

4. The Norfolk District, with the assistance of the U.S. Army Engineer Waterways Experiment Station (WES), is studying the replacement and/or extension of the Craney Island Disposal Area. WES is conducting four studies for NAO. The three studies other than the one reported herein are:

- a. Raising of the perimeter dikes at the existing Craney Island facility to el +34 ft MLW raises the question of stability of the foundation of that dike system. A report is being prepared on that subject (Fowler et al., 1986).
- b. The expected storage volumes for five alternative configurations of the extension of Craney Island to the west and/or north is the subject of another WES study (Goforth, 1986). Estimated volumes from that study were used in this report.

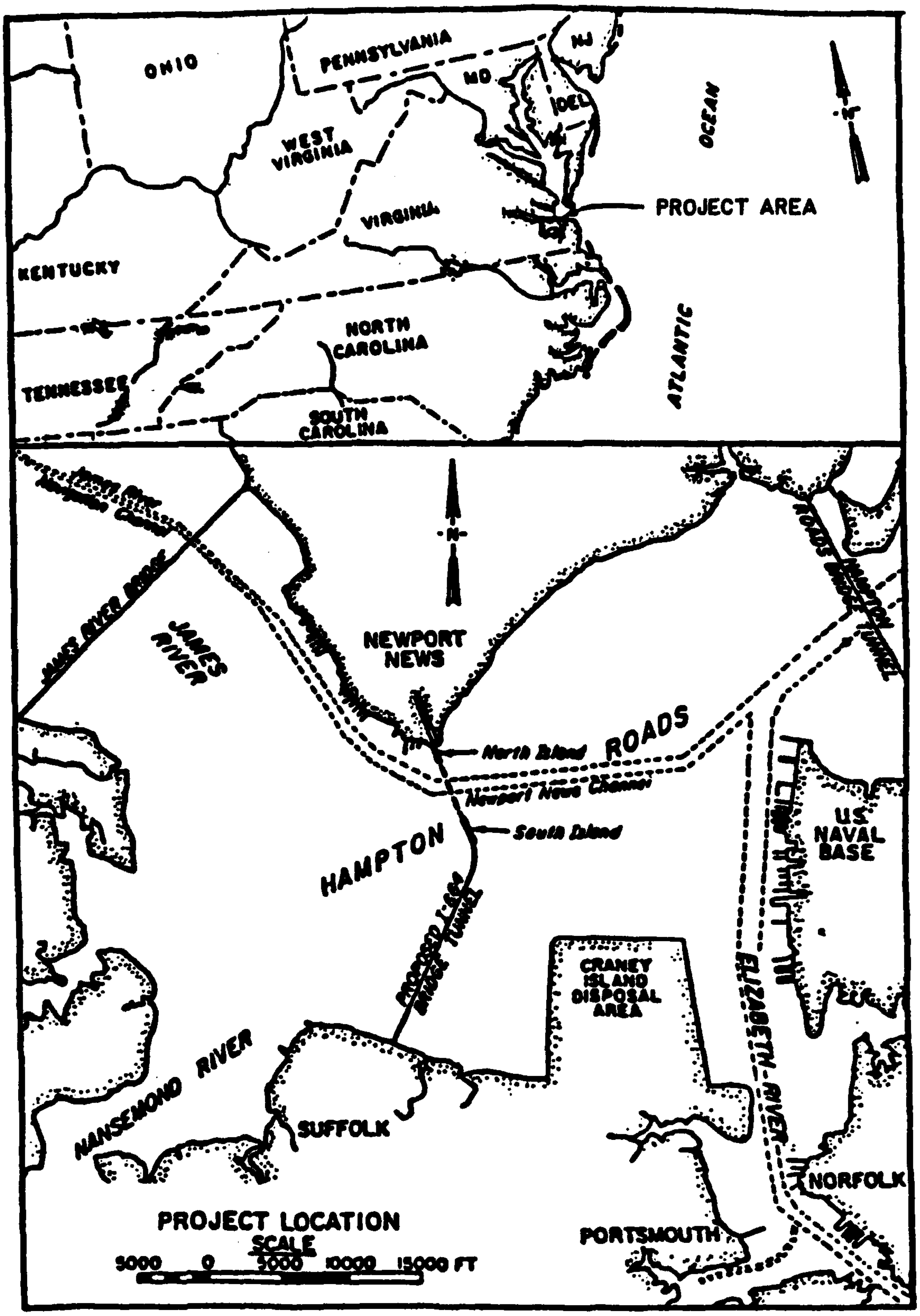


Figure 1. Project location

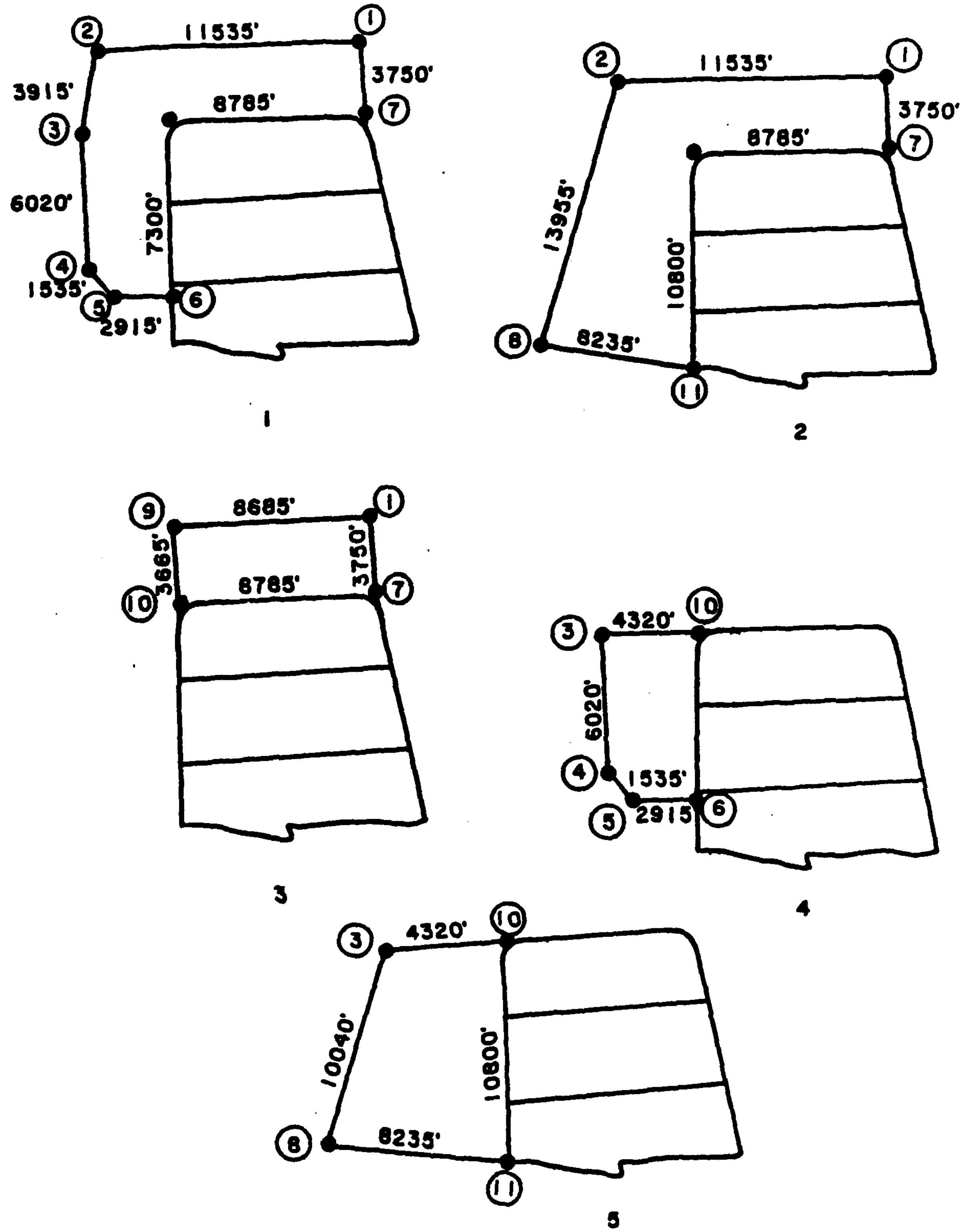


Figure 2. Alternative configurations for expansion replacement of Craney Island

c. The effects of each of the proposed five alternative configurations of the extension of Craney Island on vicinity hydrodynamics has also been studied by WES. A report by Heltzel (1986) presents the estimated changes in tidal velocities due to the proposed new configurations.

Objective

5. The objectives of the study reported herein are: (a) to evaluate the feasibility of constructing the proposed extension dikes from the geotechnical engineering standpoint; and (b) to make a cost estimate for the construction of the dike system for each of the five proposed configurations.

Scope of Work

6. The completion of the study objectives was accomplished in several steps:

a. Available geotechnical and related information was assembled from conferences with WES and NAO personnel and from existing literature, both published and unpublished.

b. A single cross section geometry and set of materials properties was adopted for slope stability analyses of the perimeter dikes. For this study, dike height was limited to el 8 ft MLW.

c. Slope stability analyses were made using a micro computer program. Because detailed subsurface investigation data is not yet available, a range of expected conditions was evaluated. Water depths and foundation soil shear strength were varied over a range to fit expected project conditions. The effects of these variables on factor of safety against sliding were evaluated. This data was then compared with available data on site conditions. Additional, upgraded comparisons can be made when more complete, updated subsurface investigation data becomes available.

d. Using navigation charts of the area, the configuration of the bottom along the proposed dike alignments was determined. Based on observation data from the existing Craney Island facility, an estimate was made of the expected subsidence of the dikes due to compression and/or displacement of the bottom soils. These values were used to determine estimated volumes for the dike materials for the alternative configurations of Figure 2.

e. A cost estimate for construction of the dikes for each of the alternative configurations was made from estimated unit costs for placement of the total volume of dike and riprap materials.

PART II: PRIOR AND CONCURRENT STUDIES

Long-Term Disposal Work Plan, 1985

7. An unpublished study entitled "Work Plan, Norfolk Harbor and Channels, Virginia, Long-Term Disposal" (Norfolk District, 1985) was prepared by NAO as part of its program of Continuation of Planning and Engineering. That report presents, in its section on Prior Studies and Reports, a discussion of two reports of importance to the project: (a) WES Technical Report EL-81-11, "Development of a Management Plan for Craney Island Disposal" (Palermo et al., 1981), which discussed the filling of the existing Craney Island facility to el +30 MLW; and (b) a draft report by WES entitled "Effects of Nofolk Harbor Deepening on Management of Craney Island Disposal Area," (WES, 1983) which recommended extension of the facility to the west to provide for the disposal of materials from the proposed deepening project.

8. The 1985 Work Plan discussed the long term disposal problem. The channel deepening project combined with continued maintenance dredging have resulted in a two fold proposed solution: (a) raise the existing dikes to permit a fill elevation to +30 ft MLW; and (b) extend the disposal area to the west. The 1985 Work Plan states... "the need for expanding and/or replacing the Craney Island Disposal Area is now critical."

9. The 1985 Work Plan recommended that several studies be made to help resolve the long term disposal problems facing Hampton Roads. Within the study area of "Engineering and Design Studies", the 1985 Work Plan report recommended "Preliminary Engineering Studies of Construction Techniques and a Construction Cost Estimate", which is the subject of the present report.

Hydrodynamic Evaluation

10. A study entitled "Norfolk Harbor Long Term Disposal Study, Virginia, Evaluation of Craney Island Enlargement Alternatives," (Heltzel, 1986), was made by the Hydraulics Laboratory, WES, of the effects of four of the proposed alternative configurations of Craney Island on the vicinity hydrodynamics in the lower James River (Hampton Roads). The four configurations used were numbers 1, 2, 3, and 4 shown in Figure 2.

11. Each of the four proposed configurations was considered with and without the presence of the proposed Newport News Island. Newport News Island is a proposed new disposal facility just east of North Island and north of Newport News Channel. Each configuration was analyzed theoretically for its effect on maximum ebb/flood velocities at three selected locations in the Newport News Channel north of Craney Island just east of the proposed I-664 Bridge Tunnel. Location No. 1 is just east of North Island. Location No. 2 is due north of the west leg of Craney Island. Location No. 3 is due north of the east leg of Craney Island. The percentage increases in ebb/flood velocities are shown in Table 1.

Table 1

Percentage Increase in Ebb/Flood Velocities

(Heltzel, 1986)

Alternative	Location No. 1 Ebb	Location No. 1 Flood	Location No. 2 Ebb	Location No. 2 Flood	Location No. 3 Ebb	Location No. 3 Flood
1	10%	14%	19%	17%	8%	4%
1-N*	10%	33%	30%	19%	9%	1%
2	11%	13%	21%	11%	9%	0
2-N	11%	28%	36%	13%	11%	0
3	4%	7%	12%	11%	11%	4%
3-N	4%	22%	23%	12%	12%	1%
4	2%	2%	3%	0	1%	0
4-N	1%	14%	11%	1%	0	0

* Alternative with Newport News Island.

12. The report concludes that Alternative No. 4, which has no northern protrusion, will cause the least impact on general circulation in the lower James River. However, none of the other configurations, especially without Newport News Island, is estimated to cause detrimental change.

Disposal Volume Study

13. Another study by WES that is currently in the draft stage presents calculations of the storage volumes to be expected for each of the five alternative configurations. The working draft is entitled "Disposal Life

Evaluation of Alternative Expansion Configurations for Craney Island Disposal Facility," (Goforth, 1986).

14. The study assumed two exterior dike el +8.0 ft MLW for the first filling, followed by extended filling at el +34.0 ft MLW. The higher elevation assumes the dredged material has consolidated to el +1.5 ft MLW and that the secondary dikes will be placed about 364 ft inboard from the centerline of the primary dikes. The report also presents suggested management guidelines. The volume estimates contained in the Goforth (1986) report are included in Table 7 of this report.

Raising of Existing Dikes

15. A study has been made by the Geotechnical Laboratory, WES, of the foundation stability of the proposed final perimeter dikes to be used to raise the existing Craney Island facility to +34 ft MLW. The study report is in working draft form and is titled "Perimeter Dike Stability Analysis, Craney Island, Norfolk District," (Fowler et al., 1986).

16. It was concluded that it is technically feasible to raise the west perimeter dike from +26 ft MLW to +34 ft MLW with a factor of safety of at least 1.20 provided sand berms are placed outboard of the higher dikes, i.e., the berms would be placed on the existing roadway, and that additional sand berms would be placed in the water adjacent to the west perimeter dike.

17. It was also concluded that it is technically feasible to raise the east and north perimeter dikes to el +40 ft MLW with a factor of safety in excess of 1.30 provided the dike crests are set back 420 to 450 ft, respectively.

1986 Subsurface Investigation

18. Seven test borings were made by NAO in late 1986 along the proposed north alignment of the expansion of Craney Island, alternative 3, Figure 2. Preliminary logs of the borings have been made available to the authors by NAO (Personal Communication, November, 1986, M. T. Byrne, Geotechnical Branch, U.S. Army Engineer District, Norfolk). The investigation included the procurement of tube samples and laboratory unconfined compression or Q-triaxial compression tests of the marine clays.

PART III: GEOTECHNICAL ENGINEERING STUDIES

Design and Construction Considerations

19. Embankment design considerations for the perimeter dikes, as given in Chapter 6 of Engineer Manual EM 1110-2-5027, "Confined Dredged Material Disposal" (Office, Chief of Engineers, 1987), include: project constraints; foundation conditions; dike stability and settlement; construction materials and methods, and erosion control.

20. Discussion of project constraints, such as time, funding, location, height, available space, environmental safety, and aesthetics, is beyond the scope of this report.

21. The major design and construction concern of this study, from the geotechnical engineering standpoint, is the structural stability of the perimeter dike. The stability is, in turn, directly dependent on the shear strength of the soft harbor bottom marine clays and the rate at which strength will increase under the load of the gradually placed fill. Foundation conditions must be established regarding soil stratification and the lateral and vertical distribution of shear strength, compressibility, and permeability, both horizontal and vertical.

22. Embankment stability and settlement are functions of foundation soil conditions, embankment geometry, and the embankment fill soils and their method of placement. Stability analyses must consider the amount and rate of change in foundation soil strength due to compression of the soil during placement of the embankment.

23. Because filling operations for the extension of Craney Island will take place in water having a depth of 0 to over 30 ft, the type of fill materials used in embankment construction, and the method of placement, will have a great effect on dike stability.

24. These topics: foundation soil conditions, embankment stability, construction materials and methods, as well as slope erosion protection and suggested instrumentation and monitoring, are discussed in the following sections.

Foundation Soil Conditions

25. A generalized north-south soil profile through the existing Craney Island Disposal Area (Palermo et al., 1981) is shown in Figure 3. The soil profile indicates a firm bottom at el -90 ft MLW consisting of dense sands and silty sands. Above the sands is a layer of marine clay. Harbor bottom in the study area for the present report is at about zero ft MLW to -32 ft MLW. Therefore, the marine clays are about 90 to 58 ft thick. For purposes of this study, the dense sand and underlying materials are considered a hard, incompressible layer.

26. Fowler (1986) presented a summary of the results of geotechnical subsurface investigations made in the area of the Craney Island Disposal Area during the period from 1944 to 1983. Additional subsurface investigations, for the proposed expansion area, have only been partially completed by NAO. These new investigations, made in late 1986, have verified that the marine clay deposits are very soft, nearly liquid, at the mud line and increase in shear strength with depth.

27. For clarity and continuity of presentation, several figures from the Fowler (1986) report are included in the present report. These are:

- a. Figure 4, Boring Location Plan, showing the locations of test borings made during the 1971, 1981, and 1983 subsurface investigations of the existing Craney Island. The locations of the 1986 series of preliminary borings has been added.
- b. Figures 5, 6, and 7, Shear Strength Profile, Existing East, North, and West Dike, respectively, displaying the combined results of field vane shear tests and laboratory triaxial compression tests of tube samples from the 1971, 1981, and 1983 subsurface investigations of Figure 4.

28. The marine clay layer is a continuous stratum of recent marine sediments which are presumed to be normally consolidated, i.e., they have never experienced a greater load than that imposed by their own weight. Therefore, it would be expected that soil unit weight would be lowest (void ratio highest) at the present harbor bottom (profile line) and increase (void ratio decrease) with depth. Concomitantly, shear strength should increase with depth. Assuming a relatively constant type of clay mineral, the actual shear strength may vary from point to point as a result of natural variations in sand and/or silt content

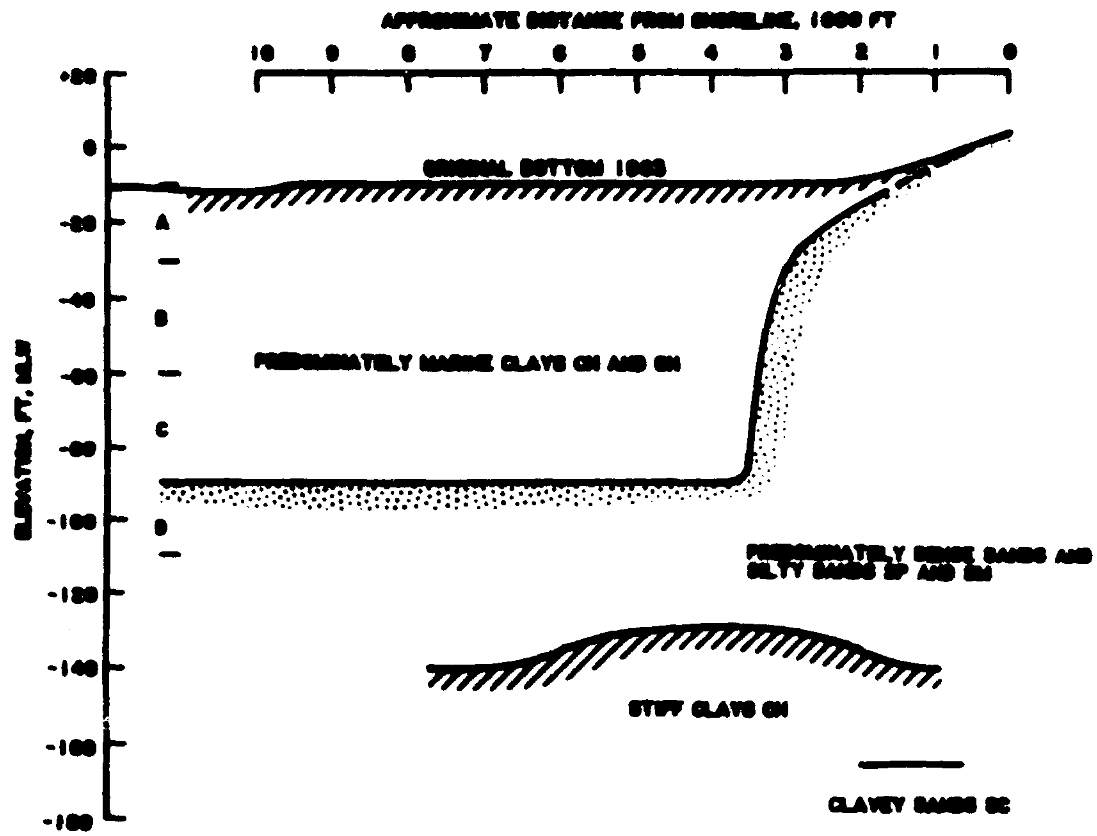


Figure 3. Generalized subsurface profile within the matrix

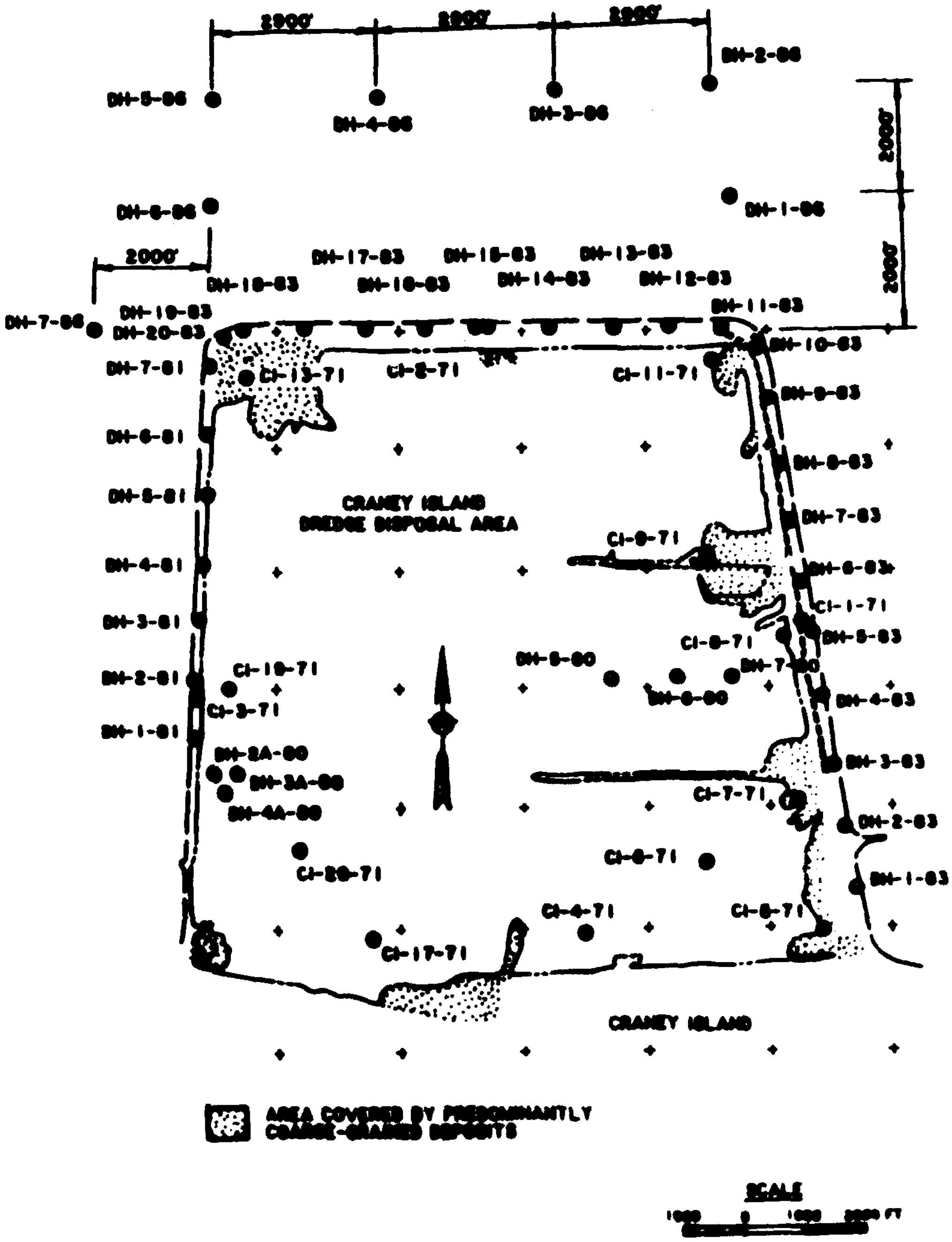


Figure 4. Boring location plan, 1971-1986 investigations

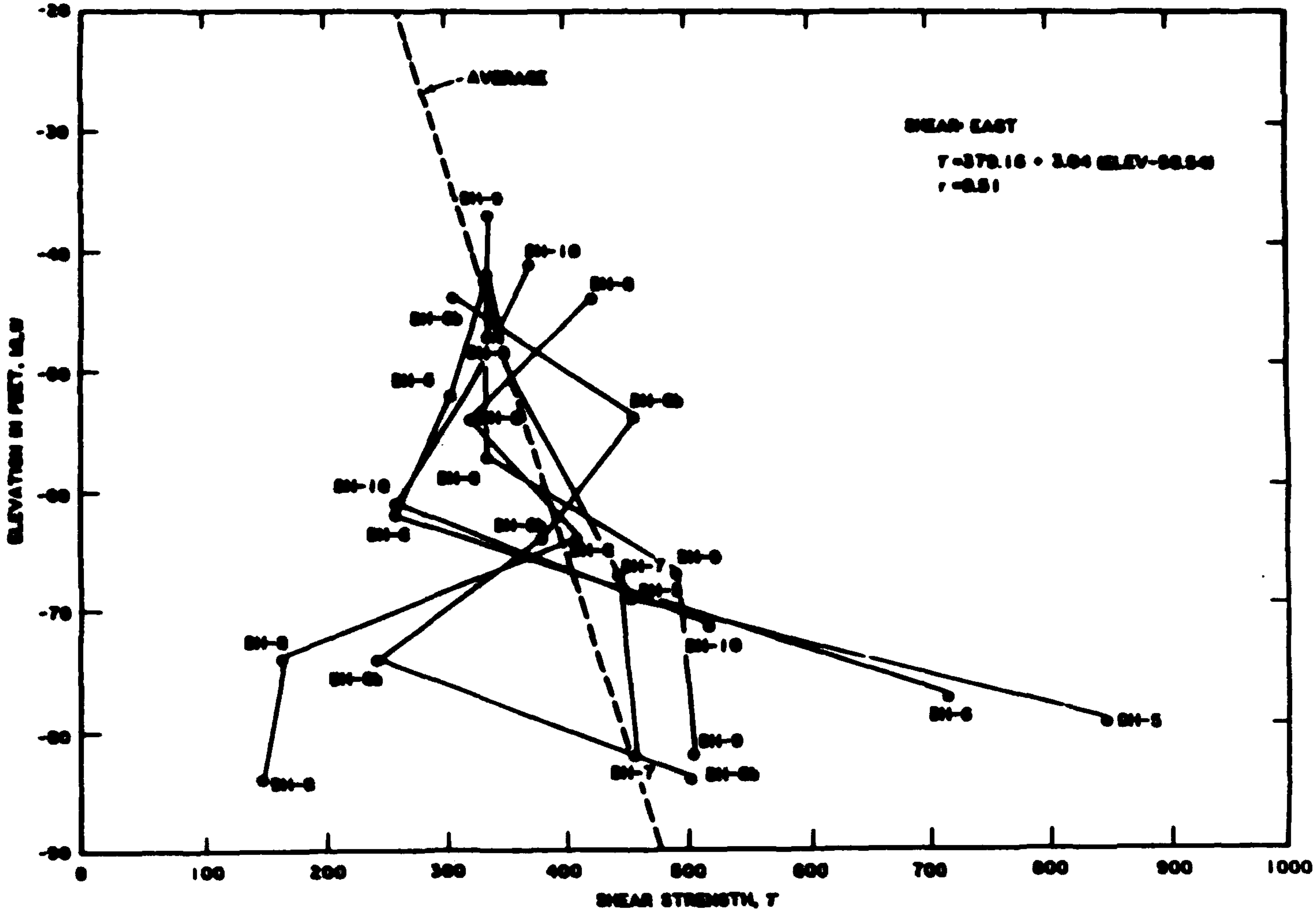


Figure 5. Shear strength profile, existing east dike

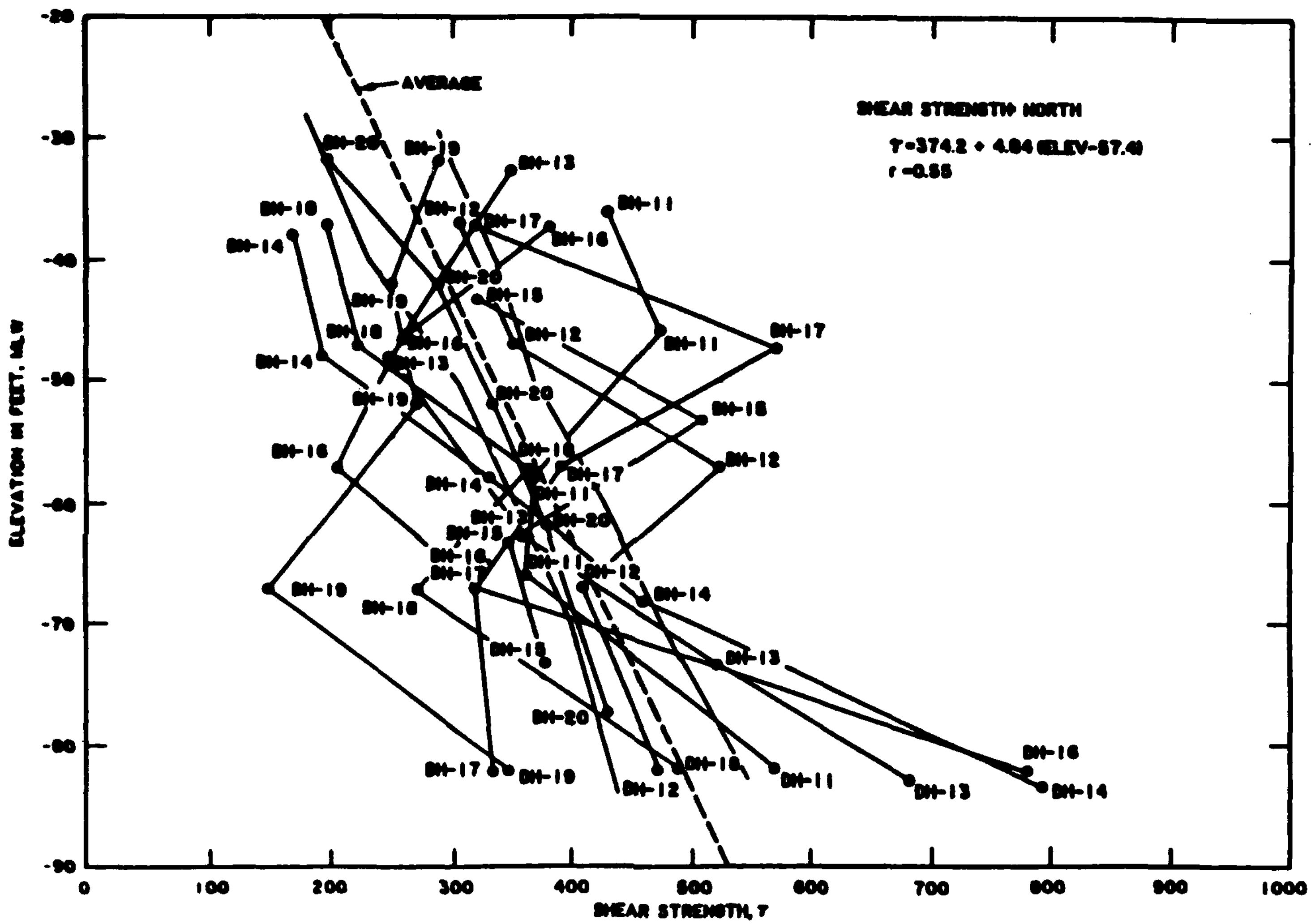


Figure 6. Shear strength vs depth-north

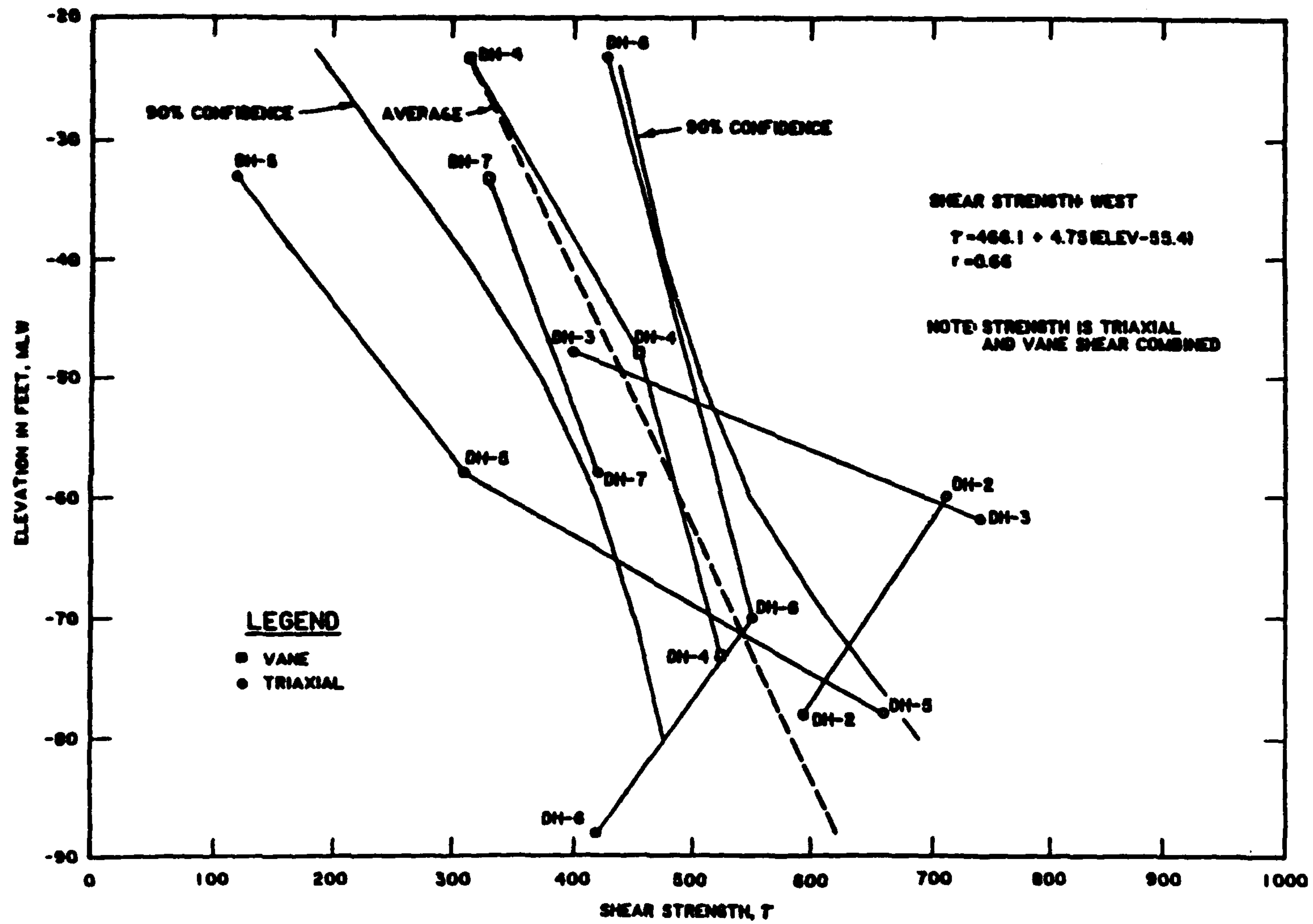


Figure 7. Shear strength vs depth-west

clay, with variation in the consolidating pressure from overlying soils, and with the organic content. The Atterberg limits, i.e., liquid limit and plasticity index, will vary directly with the amount of clay mineral and the amount and type of organic matter.

29. The gradual placement of the weight of the sand fill should have the effect of densifying the soil, with its concomitant strength increase, at a rate that will permit the soil to support the load without failure. A statistical relationship has been found between the undrained shear strength (cohesion) of saturated clays, the effective overburden pressure, and the plasticity index of the clay. This the Skempton (1948) c/p ratio in which the cohesion, $c = p'(0.11 + 0.0037\ PI)$, where p' = effective overburden pressure and PI = plasticity index.

30. The extent to which this effect may be expected to occur may be seen by reviewing data from the existing Craney Island facility. A group of tube sample borings was made in 1948-49 along the perimeter dike area prior to construction. The General Design Memorandum (Norfolk District, 1953, page I-5) stated ..."The initial exploration consisted of five drill holes numbered 22 through 26 spaced approximately 5,000 feet apart along the dike centerline ..." and ..."A second series of drill holes, numbered 81 through 86, located along the dike centerline and spaced midway between the previous five holes...". In an internal memorandum regarding the proposed raising of the Craney Island dikes (Report of Soil Testing, 20 November 1972, Norfolk District, Corps of Engineers) the NAO recorded the results of tube sample tests from three borings made in 1971, CI-1, CI-2, and CI-3, including unconfined compression tests. The locations of these borings are shown on Figure 4. The memorandum compared samples from the 1948 and the 1971 borings from about the same elevation in borings at about the same location. The only data that was amenable to direct comparison regarding consolidation effects was the dry unit weight of the tube samples. This comparison is shown in Table 2. Although this data is not definitive, the comparisons do indicate a general increase in dry unit weight after placement of the dikes. Presumably, this also indicates an increase in shear strength.

31. Figure 8 combines data from the 1949 and 1971 subsurface investigations, which were both made along the centerline of the now-existing dikes. The 1949 borings, 81, 82, 83, and 84, included tube samples and unconfined compression tests. Samples from the three borings from the 1971

series, CI-1, CI-2, and CI-3, also had unconfined compression tests. Taken as a group, the least squares best fitting line to the 1949 test data indicated: (a) the shear strength at the mud line was nearly zero; (b) the average cohesion at el -90 MLW was 177 PSF; and (c) the cohesion increased with depth at an average

Table 2

Comparison of Soil Unit Weight From 1948 and 1971 Test Boring Samples

Drill Hole No.	Year Made	Sample No.	Elevation Feet, MLW	Dry Unit Weight, PCF
22	1948	3	-31 to -33	50.2
CI-1	1971	U-9	-26 to -28	59.8
22	1948	10	-72 to -74	48.1
CI-1	1971	U-16	-61 to -63	51.3
23	1948	6	-31 to -33	46.4
CI-3	1971	U-10	-35 to -37	58.0
23	1948	10	-47 to -49	48.4
CI-3	1971	U-12	-43 to -45	51.4
26	1948	2	-29 to -31	55.5
CI-2	1971	U-9	-31 to -33	59.9
26	1948	12	-84 to -86	57.6
CI-2	1971	U-18	-72 to -74	52.4

rate of about 1.9 PSF per foot of depth. The 1971 data in Figure 8 shows a significant increase in shear strength with consolidation of the clays under the weight of the dikes. Further comparison of "before and after" shear strength is obtained by comparing the 1949 borings of Figure 8 with the data of Figures 5, 6, and 7 for the 1981 to 1983 subsurface investigations. All of the data from 1971 to 1983 indicates an average shear strength at el -15 MLW of 170 to 288 PSF and an average shear strength at el -90 MLW of 293 to 630 PSF, with a rate of increase 3.0 to 4.8 PSF per foot of depth. Some of the data of Figures 5 and 6,

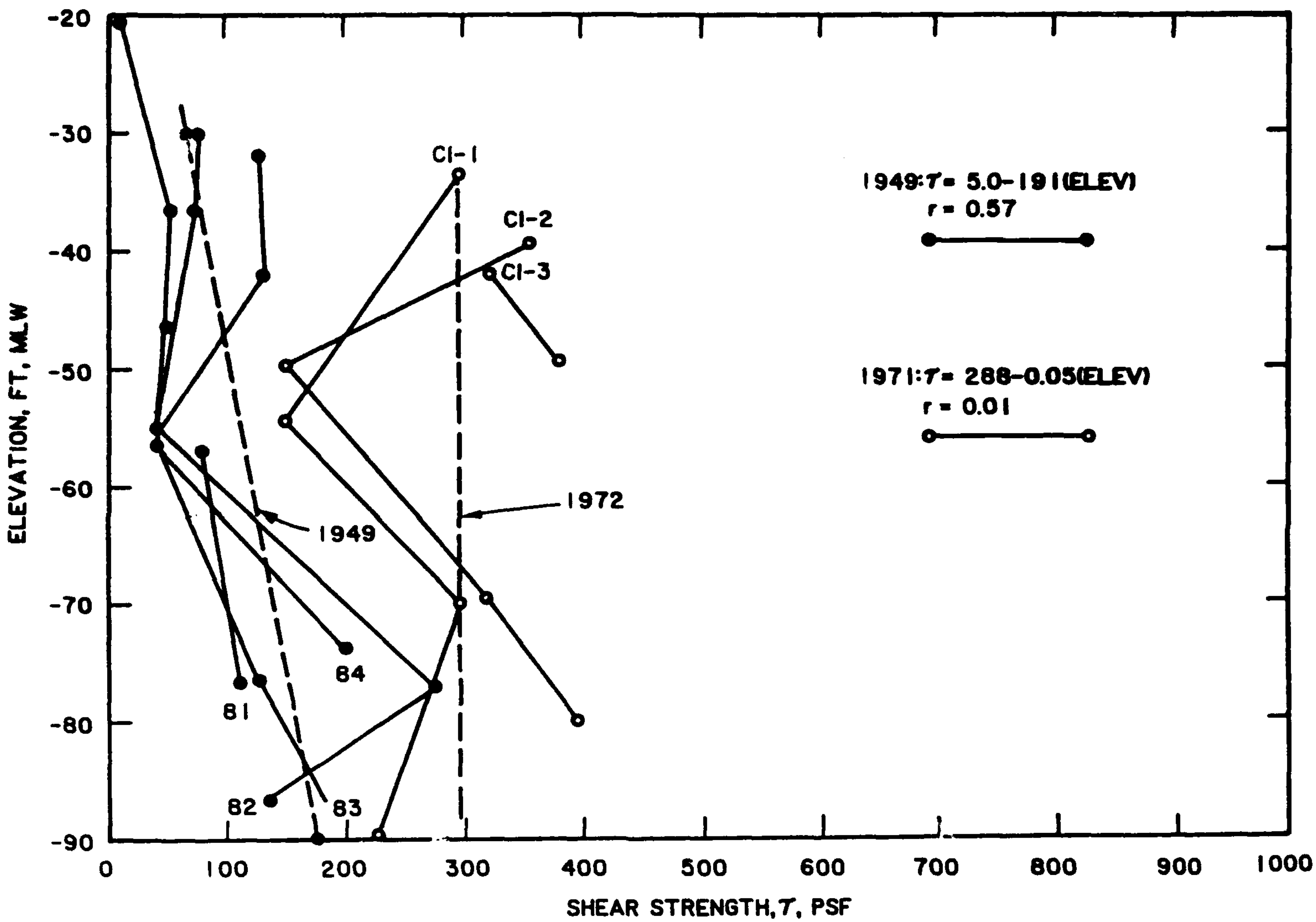


Figure 8. Shear strength profile, 1949 and 1971 borings

and all of the data of Figure 8, tend to show a soft zone at about el -50 to -60 MLW, which causes the Figure 8 data to havea lower rate of increase with depth than the data of Figures 5, 6, and 7. This factor cannot be fully evaluated with the available data.

32. Figure 9, Shear Strength Profile, 1986 Borings, is adapted from data provided to the authors by NAO, described above. An average shear strength, from the least squares analysis, at the average mud line at el -20 MLW is 50 PSF. The average shear strength at el -90 MLW is 393 PSF, with a rate of increase of about 4.9 PSF per foot of depth. This indicates a somewhat stronger profile than the 1949 data of Figure 8.

33. Using the c/p ratio relationship with an average PI of 35 and a submerged unit weight of 28 PCF, the strength vs depth relationship for a water depth of 20 ft indicates a cohesion of zero at the mud line and cohesion at the bottom of the layer of about 470 PSF. This compares well with the data of Figure 9. A similar analysis for an existing dike in 15 ft of water, using a submerged unit weight of 63 PCF for the sand, gives a cohesion of 225 PSF at the top of clay and a cohesion of 730 PSF at the bottom, with a rate of increase of 6.7 PSF per foot of depth. This is somewhat higher than the data of Figures 5, 6, and 7. Either the assumptions used for average PI and/or average submerged unit weight are too high or the clay under the existing dikes has not yet reached full consolidation.

34. Data from the laboratory tests of samples from the subsurface investigations referenced above, i.e., 1948-49 and 1971, indicate:

a. Atterberg Limits: Liquid limit ranges from 40 to 105, with typical values of 60 to 90. Plastic limit ranges from 25 to 40, with typical values of 30 to 35. Plasticity index ranges from 15 to 65, with typical values of 30 to 60.

b. Wet unit weights range from 85 to 102 lbs per cu ft. Prior to compression of the clay under dike pressure (1949 data), the void ratios ranged from over 3.00 near the mud line to about 2.20 at depth. Void ratios after dike placement (1971 data) range from about 2.75 near the profile line (mud line) to about 1.50 at depth. The lower void ratio values (increased density) reflect compression of the clay by its own weight and by the additional weight of the dikes.

c. Compression indexes from laboratory consolidation tests of undisturbed samples ranged from 0.55 to 0.90. A compression index value of 0.58 was used in volume estimate calculations by Goforth (1986).

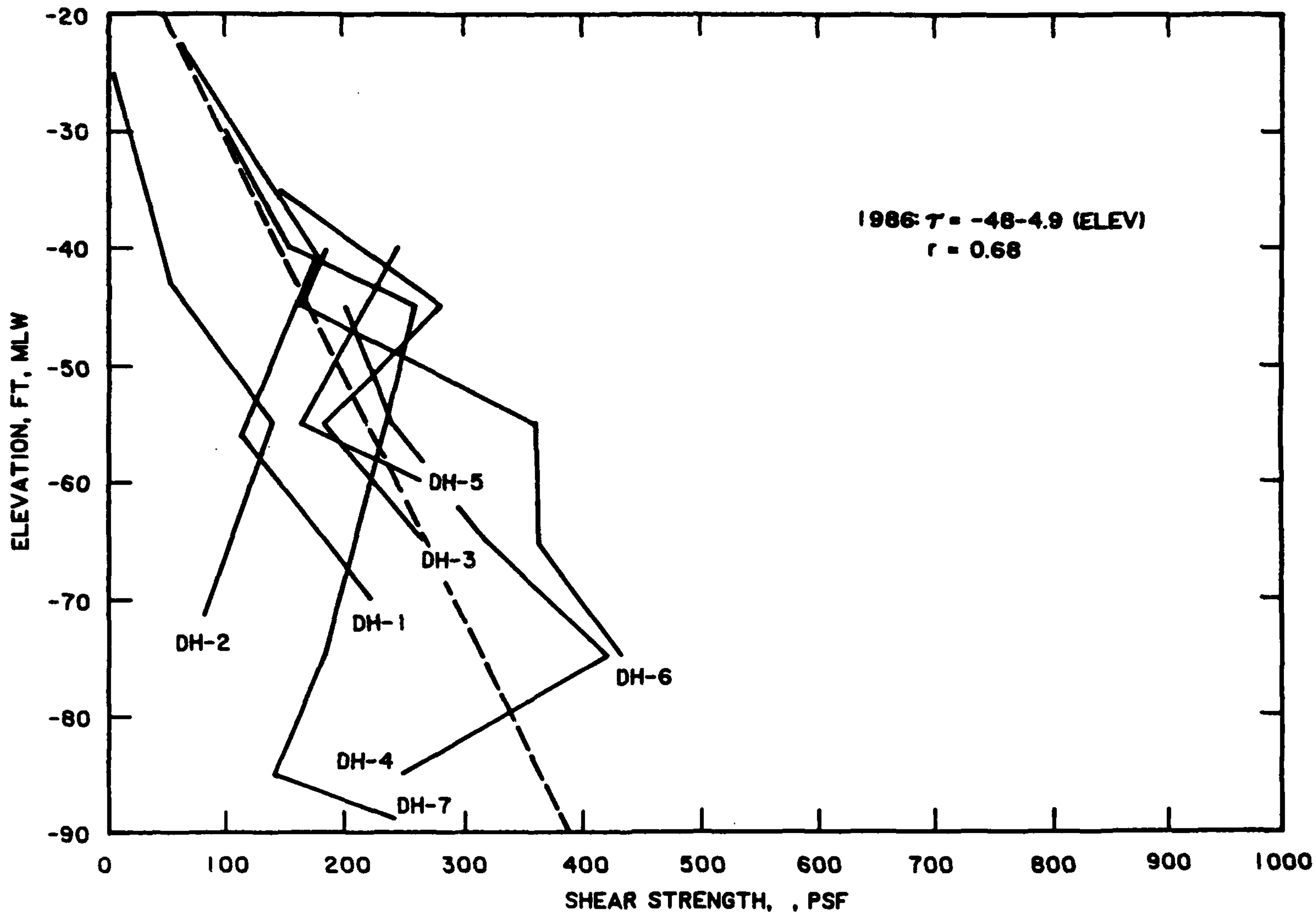


Figure 9. Shear strength profile, 1986 borings

Embankment Stability

35. The existing shear strength profile along the proposed dike alignments is not well known, although the 1986 subsurface investigation is of value. The consolidation of the marine clays under the embankment load is expected to increase the present in situ strengths to values similar to those found under the existing Craney Island dikes. However, the time rate at which this will occur is also unknown. This is partially dependent on the rate of construction of each segment of dike, and the construction rate has not been established. The amount of lateral displacement of soft muds is also unknown. In such a situation, stability analyses can be made only for assumed conditions. From analyses made for a reasonable range of governing parameters, the conditions needed to achieve a factor of safety of 1.3, or any other factor of safety as required, can be determined. Then, the design and/or construction considerations needed to achieve those conditions can be established.

36. Embankment Cross Section. The embankment cross section used for slope stability analyses is shown in Figure 10, which was adapted from the cross section used in the initial Craney Island perimeter dikes (Palermo et al., 1981) and was used in a concurrent study of storage volumes (Goforth, 1986). The essential elements of the calculation cross section are:

a. The dike crest is at +8.0 ft MLW and is 22 ft wide.

b. The interior above-water slope is 1V:2H.

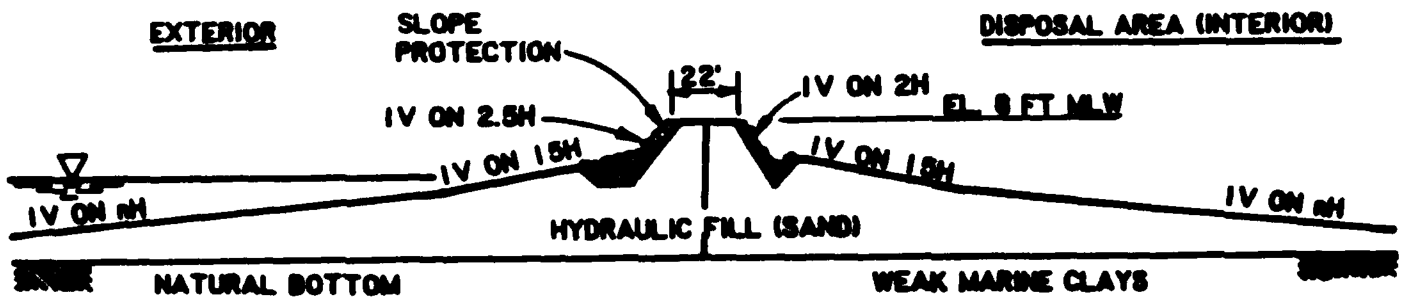
c. The exterior above-water slope is 1V:2.5H.

d. A transition slope of 1V:15H is used on both sides of centerline between +4.0 ft MLW and -2.0 ft MLW.

e. A very flat underwater fill, with a slope of 1V:30H, extends on both sides of centerline from the transition slope at -2.0 ft MLW to the natural bottom. Hydrographic surveys of the underwater portion of the existing Craney Island perimeter dikes showed slopes of 1V:30H or flatter. Hydrographic data indicate the maximum water depth expected in any of the alternative configurations is 32 ft.

f. The exterior and the interior slopes from +8.0 ft MLW to -2.0 ft MLW will be covered with stone riprap as wave erosion protection. A longitudinal trench will be dug on both upstream and downstream slopes at the intersection of the steep dike slope and the transition slope to act as the base for riprap slope protection.

37. Embankment Materials. It is assumed that the entire embankment, above and below water level, will be made of sand. The sand below water level will be



(NOT TO SCALE)

NOTE: (a) UNDERWATER FILL SLOPE VARIES 1V:30H TO 1V:70H.
(b) MAXIMUM WATER DEPTH ALONG THE DIKE ALIGNMENT IS 34 FT.

EXTERIOR SLOPE-RIPRAP, 24 IN. THICK WITH A NOMINAL STONE SIZE OF 1000 LB.
PROTECTION-BEDDING LAYER, 9-IN. THICK CRUSHED ROCK; GEOTEXTILE.

INTERIOR SLOPE-RIPRAP, 12-IN. THICK WITH A NOMINAL STONE SIZE OF 200 LB.
BEDDING LAYER, 9-IN. THICK CRUSHED ROCK; GEOTEXTILE.

Figure 10. Typical dike cross section used for slope stability studies

placed hydraulically, either by dredging discharge or bottom dump barges. The material properties used in the computer analyses are shown in Table 3. These are the same or similar properties used in other studies (Fowler, 1986). Strength properties for the marine clay layer were varied for the several stability analyses.

Table 3

Material Properties Used in Slope Stability Analyses

Material	Unit Weight lb/cu ft	Type	Shear Strength CPROFL	RATEIN	Phi Angle
Moist Embankment Sand	110	Conventional Shear Str.	0		30
Saturated Embankment Sand	125	Conventional Shear Strength	0		30
Consolidating Dredged Material	90	Linear Increase w/Depth	35	1.8	0
Soft Marine Bottom Clay	100	Linear Increase w/Depth	200 250 300 350	2.5 5.0	
Dense Base Sand	125	Conventional Shear Str.	0		30

38. Stability Analysis Computer Program. A microcomputer program, UTEXAS2, Version 1.109, was used in slope stability calculations for a number of cross section configurations for the proposed perimeter dikes. The program is described in the draft WES Instructional Report GL-86 entitled "User's Guide: UTEXAS2 Slope Stability Package; Volume 1, User's Manual" (CAGE,G-CASE Task Group on Slope Stability, 1986). A typical UTEXAS2 computer program input is shown in Appendix A.

39. The wedge method of analysis (noncircular) was used in preference to the circular arc method. A number of analyses were first made, with the geometry and materials properties described above, using the UTEXAS2 program in Circular Arc (automatic) Search mode. In all cases tried, numerous error messages appeared indicating that the portion of the failure arc entering and exiting the

embankment surface was at too steep an angle, not utilizing the shear strength of the sand, and indicating an invalid factor of safety that is far too low. The critical failure circles had centers just above or just below the outside toe of the above-water slope. The effect was that of a shallow bearing capacity failure of a footing (the above-water dike) over a sand layer underlain by a soft clay layer.

40. In the sand embankment, the critical failure surface should approximate an active pressure surface at an angle of about 600 from the horizontal at the driving end and a passive pressure surface at an angle of about 300 from the horizontal at the resisting end. The bottom of the central portion, between the active and passive wedges, may be a horizontal line in the softest part of the clay layer, or it may be an arc of a circle extending downward into the clay layer at some depth, or it may be some form of noncircular surface between these limits.

41. The geometry of the assumed failure surface for the wedge analyses is shown in Figure 11. Points 1 and 2, in the passive pressure wedge, were allowed to move horizontally only. The maximum passive surface angle permitted was 400 from the horizontal. Points 3 and 4, in the central failure zone, were allowed to move vertically during the automatic search so that any geometry from a horizontal line to a circular arc could be approximated. Point 5 was fixed at a position where the greatest active pressure could be generated. Point 6 was allowed to move horizontally to establish the active pressure surface.

42. Slope Stability Analyses. Thirty six slope stability analyses, using the wedge method, were made for this study: 3 values of CPROFL, 3 values of RATEIN, and 4 values of water depth. All analyses were for the end of construction condition, i.e., the full embankment is in place.

43. The assumed values of undrained shear strength at the surface of the marine clay layer, expressed as "cohesion at the profile line" (CPROFL), were 100, 200, and 300 lbs per sq ft. The values for rate of increase (RATEIN) of cohesion with depth were zero, 2.5, and 5.0 lbs per sq ft per ft of depth.

44. Water depths of 5 ft, 15 ft, 25 ft, and 35 ft from mean low water to the natural harbor bottom were used. This range of values was determined from National Oceanic Atmospheric Administration (NOAA) charts of the area.

45. Results of the slope stability analyses using the various criteria given above are presented in Table 4.

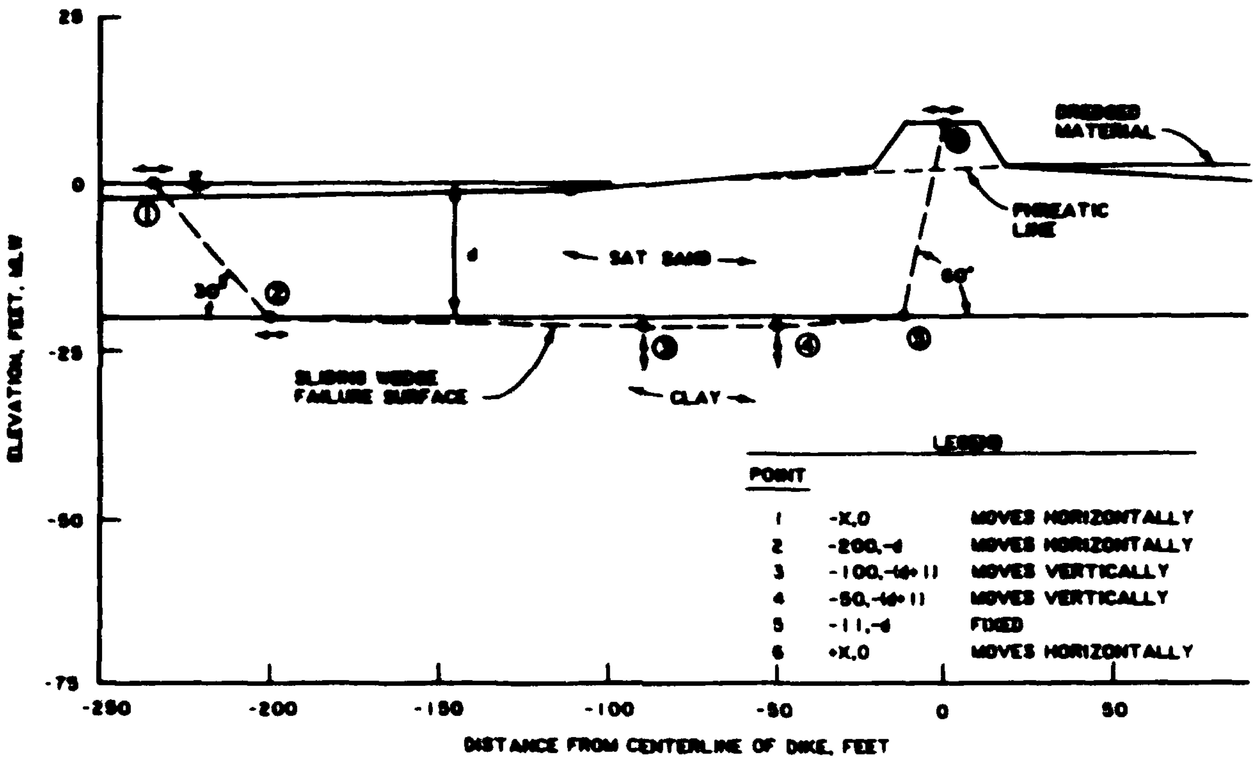


Figure 11. Assumed failure surface used in wedge analyses

Table 4

Results of Slope Stability Analyses

Profile Depth Ft. MLW	Cohesion at Profile Depth CPROFL, PSF	Rate of Increase of Cohesion with Depth RATEIN, PSF/FT	Factor of Safety
-05	100	0.0	.971
-05	100	2.5	1.359
-05	100	5.0	1.571
-05	200	0.0	1.846
-05	200	2.5	2.220
-05	200	5.0	2.464
-05	300	0.0	2.698
-05	300	2.5	3.031
-05	300	5.0	3.250
-15	100	0.0	.911
-15	100	2.5	1.330
-15	100	5.0	1.550
-15	200	0.0	1.650
-15	200	2.5	2.000
-15	200	5.0	2.110
-15	300	0.0	2.323
-15	300	2.5	2.582
-15	300	5.0	2.675
-25	100	0.0	1.155
-25	100	2.5	1.656
-25	100	5.0	1.927
-25	200	0.0	1.887
-25	200	2.5	2.234
-25	200	5.0	2.557
-25	300	0.0	2.446
-25	300	2.5	2.760
-25	300	5.0	2.970
-35	100	0.0	1.323
-35	100	2.5	1.743
-35	100	5.0	2.164
-35	200	0.0	2.469
-35	200	2.5	2.857
-35	200	5.0	3.229
-35	300	0.0	3.539
-35	300	2.5	3.922
-35	300	5.0	4.292

46. Discussion. The effect of three variables on factor of safety was considered in the stability analyses: cohesion at the profile line (CPROFL), rate of increase of cohesion with depth (RATEIN), and depth of water at mean low water level. Figure 12 contains graphs of the data of Table 4, with separate graphs of factor of safety vs CPROFL for each of the four water depths studied.

47. The most significant variable in the determination of factor of safety was the cohesion at the profile line (CPROFL). The factor of safety increased from 0.75 to 1.1 times for each 100 PSF increase in strength. The rate of increase of shear strength with depth (RATEIN) also significantly affected factor of safety. When the RATEIN was zero, points 3 and 4 of Figure 11 extended almost to the bottom of the marine clay layer, approximating a circular arc failure to mode. As RATEIN increased to 2.5 PSF/FT, points 3 and 4 rose higher, and at 5.0PSF/FT rose almost to the surface of the clay layer, more closely approximating a sliding wedge failure, causing the active and passive wedges to move further apart. Variation in depth to natural bottom caused a slight increase in factor of safety as the thickness of the sand embankment (depth) increased greater than about 15 ft. This effect occurs because of the greater tolerance of the thicker sand layers to the non uniform loading of the dike configuration.

48. All evidence, including the data of Figures 8 and 9, indicates that RATEIN of zero is not to be expected. Rather, RATEIN ranged from 1.9 to 4.9 PSF/FT in the 1949 and 1986 subsurface investigation data. Therefore, all that is needed to support the design embankment at shallow depths, about 15 ft or less, is a CPROFL of about 100 PSF at the end of construction. As the depth becomes greater than 15 to 20 ft, the needed value drops to 70 PSF. Cohesive strength of this magnitude is nearly present in the clay layer under the deeper waters before placement of the embankment. With gradual placement of the sand, consolidation settlement and displacement of the very soft muds should provide a very safe supporting environment for the embankments placed in waters over about 15 ft deep.

49. Assuming that clay shear strengths are the same for the new embankments as is shown in Figures 8 and 9, then for embankments under 15 to 20 ft thick a factor of safety of 1.3 against failure depends on increasing the CPROFL strength to at least 100 PSF by the end of construction by means of displacement of the softest muds followed by consolidation under the embankment load. The rate of

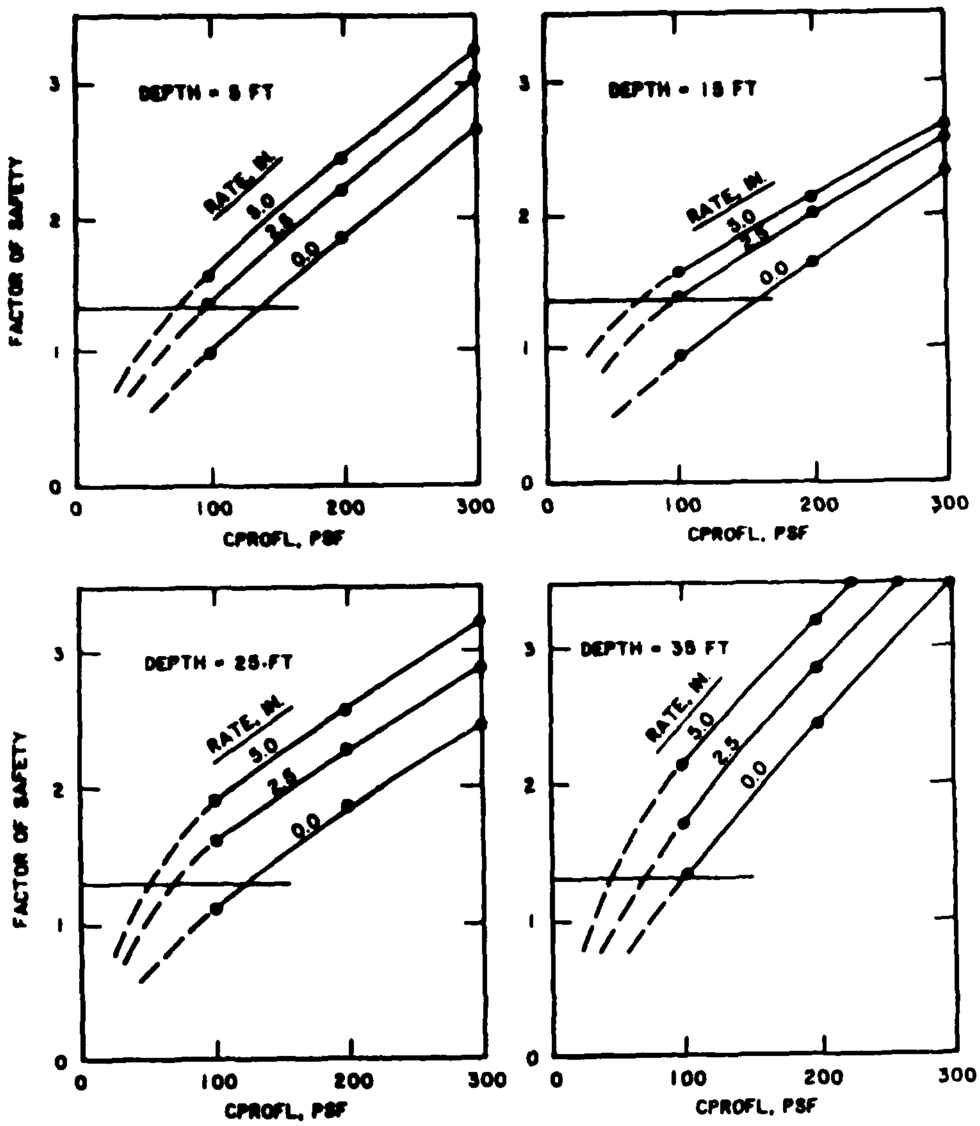


Figure 12. Slope stability factor of safety related to cohesion at profile line (CPROFL)

loading needed to achieve this cannot be determined from existing data. However, these were precisely the conditions under which the existing Craney Island Disposal Area perimeter dikes were placed. The fact that they were successfully placed indicates that the required increase in strength of the clay will, in fact, be achieved. Therefore, the rate of placement of the sand embankment in the shallower waters, perhaps less than 15 to 20 ft deep should be limited to that used in the original facility construction. Although the thicker sand embankments can be sustained by a lower strength clay, the rate of placement of these embankments must also be limited. In the early phases of their placement, they also have the smaller thickness of the embankments placed in the shallower waters and must develop a higher CPROFL to remain stable.

Construction Materials and Methods

50. Consideration should be given to the following factors in the design and construction of the perimeter dikes:

a. Nearly all of the volume of the dikes is below Mean Low Water elevation. To cause rapid sedimentation, only coarse grained materials, i.e., sand and gravel, should be permitted in the underwater portion of the dikes. The fill should have no more that 20 percent finer than the No. 200 sieve. It is desirable that the same material be used in the above water portion of the dikes. The sandy fill is expected to come from normal maintenance dredging of the channel.

b. Geotextiles (permeable textile materials) are not useful to reinforce the strength of the dikes. The mode for slope stability failures is shown on Figure 11. The dikes do not tend to spread at the base. Instead, the dikes will tend to move as a unit as part of a deep seated movement of the entire embankment.

c. Geotextiles, in the form of filter cloth, will be of benefit for the riprap as the contact between the surface of the dike and the overlying crushed stone bedding layer. This will obviate the need for a graded filter.

d. The placement of granular fill for the underwater sections of the embankments will require close control. It is assumed the sand will be placed either by direct discharge from a dredge or will be placed by bottom dump barges. It will be absolutely necessary that the fill be placed uniformly in relatively thin layers over the entire fill area. The thickness of the layers should be very small at first since the weight of this fill must cause densification, and strength increase, of the upper zone of the clay layer that is nearly liquid in behavior. Once consolidation

has started, then the fill layers may get progressively thicker or they may be placed at a faster rate. The objective is to prevent overloading the soft soils, particularly by nonuniform loading, to the extent that a stability failure occurs.

e. During fill placement, continuous depth soundings should be made on pre-established survey lines. Excessively non-uniform lifts should result in mud waves, or shallow slope failures. These are similar to bearing capacity failures for continuous footings. Therefore, rapid changes in elevation, up and/or down, are indicative of excessive mounding of fill.

f. Compaction of the underwater portion of the fill will not be possible. Compaction of the above water segment need not be any greater than machine compaction, i.e., uniform coverages of the wheels or tracks of the earthmoving equipment.

Slope Erosion Protection

51. Because of the necessity to use hydraulic fill sand for the dikes, protection from the erosive action of waves is imperative. Palermo et al., (1983) recommended the use of stone riprap on both faces of the dike, i.e., the inside as well as the outside, since erosive waves can occur during hydraulic fill placement. As indicated in Figure 10, it is recommended that:

a. Exterior slope protection consist of a 24 inch thickness of stone riprap, with nominal 1000 lb stones, over a bedding layer of 9 inches of crushed rock, placed over a continuous, overlapped geotextile (filter cloth).

b. Interior slope protection consist of a 12 inch thickness of stone riprap, with nominal 200 lb. stones, over a bedding layer of 9 inches of crushed rock, placed over a continuous, overlapped geotextile (filter cloth).

52. Riprap on both sides of the dike exposed to standing water should extend from a ditch at or below -2.0 ft MLW to the top of dike at +8.0 ft MLW.

Instrumentation and Monitoring

53. As a means of monitoring the structural behavior of the dike system during and after filling operations a number of "instrumentation groups", each consisting of a settlement monument and four or five piezometers, should be uniformly spaced along the length of the perimeter dike.

54. The settlement monuments will serve to indicate the time rate of settlement. They will also be of value to indicate amount and rate of

consolidation when the decision is made to raise the dike level of the extension, as was done for the original Craney Island facility. The piezometer data should indicate the degree of consolidation settlement that has occurred under a given loading which will provide information regarding whether the fill is being placed too rapidly.

55. Settlement monuments should be located at the outside upper edge of the dike. This will place the devices away from the area of placement of riprap and also out of the way of construction traffic on the dike center. One instrument group every 3000 to 5000 ft should suffice. The locations for instrument groups must be clearly marked and protected from construction operations. Because they will be installed before completion of the dike, it will be necessary for the dredging contractor to be aware of their locations and, if feasible, be made partially or wholly responsible for their safety and operation.

56. Each settlement monument may consist of a large steel plate, about 10 ft square, placed in dike fill a few feet above channel bottom (profile line) or may be a steel pipe placed or augered in a drilled hole extending below the fill bottom. If the steel plate is used, it can be placed on the surface of the fill after about three to five feet of fill has been placed. If the embedded standpipe is used, the embedment must be firm over the lowermost five to 10 ft of pipe. In either instance, vertical sections of steel pipe should be attached to the monument and the pipe extended upward to a short distance above MLW. The standpipe should be clearly flagged to inhibit construction damage. Filling should be done carefully in the vicinity of the monument standpipes. Periodic level surveys should measure the elevation of each monument by determining the elevation of the top of the pipe. When the fill reaches the top of the standpipe, additional sections of pipe must be added until the final dike elevation is reached. Each time a new section of pipe is added, it is imperative that "before and after" level readings be made.

57. The piezometers should be evenly spaced vertically within the marine clay deposit, one at mid-thickness, one each at about 10 percent and at 25 percent of the thickness from the bottom and top of the layer. With this, or a similar vertical spacing, the vertical distribution of porewater pressure can be established for the monitoring of consolidation progress. If possible, the

piezometer standpipes should be in the same locations as the settlement monument standpipes, perhaps rising through a hole in the steel plate. However, the settlement standpipes must be free to move downward independently of the piezometer standpipes.

58. An organized and executed plan for placement, observation and measurement, and documentation of data from the instrument groups is essential. If the recorded data is not provided to the design and/or construction groups in a timely manner, then the instrumentation will be valueless.

PART IV: COST ESTIMATE

Estimates of Dike Subsidence

59. The subsidence of the proposed dikes from the sole effect of consolidation settlement may be calculated using standard techniques of geotechnical engineering. The following assumptions were made: the clay is normally consolidated, has an average submerged unit weight of 38 PCF, a compression index of 0.58, and an initial void ratio of 2.0; the embankment sand has a submerged unit weight of 63 PCF; and all layers below the marine clay are incompressible.

60. Fowler (1986) reports measurements of subsidence of the existing Craney Island dikes. Calculations using a dike height of 18 ft result in an estimated consolidation settlement of 6.6 ft. Actual measured subsidence of the east, north, and west legs averaged 17 ft, 13 ft, and 11 ft, respectively. The upper several feet of the river bottom is expected to be so soft that it will be squeezed out laterally under the load of the sand fill. It is expected that similar or more severe displacements will occur under the extension dikes in the deeper water areas to the north and west. Based on the existing data, a very rough estimate may then be made of the amount of displacement subsidence that may be expected under the various thicknesses (water depths) of dike fills for the case of top of dike at el +8.0 ft MLW. Subsequent raisings of the extension dikes will cause additional consolidation and displacement subsidence. The estimated total dike subsidence for a series of water (embankment) depths is given in Table 5.

Estimates of Dike Material Volumes

61. The centerlines of each of the proposed dike alignments for the five alternative configurations were plotted to scale on NOAA navigation charts. A profile of centerline depths was made as a series of straight lines connecting points of depth measurements that were recorded on the charts.

62. At any given point along the centerline, the dike fill cross section consisted of two elements, the section above existing bottom and a section representing the subsidence. The above bottom section used was that of

Table 5

Estimated Total Dike Subsidence

Bottom depth, ft, MLW	5	10	15	20	25	30	35
Consolidation settlement, ft.	1.1	2.2	3.2	4.0	4.5	5.0	5.2
Displacement subsidence, ft.	1.5	3.0	4.5	6.0	7.5	9.0	10.5
Total estimated subsidence, ft.	2.6	5.2	7.7	10.0	12.0	14.0	15.7

Figure 10. The subsidence was expected to be an inverted triangle, with the two ends coinciding with the intersection of the fill with the natural bottom and with the peak of the triangle at the dike centerline. The depth of the inverted peak was estimated from the values given in Table 4. Using average end areas, estimated dike volumes for each of the individual dike segments shown in Figure 2 are given in Table 6.

Table 6

Estimated Dike Material Volumes for Various Proposed Dike Segments

Proposed Dike Segment From Point	To Point	Length, Ft.	Volume, Cu. Yds.
1	7	3750	1186790
1	2	11535	10748600
2	3	3915	308633
3	4	6020	1802570
4	5	1535	359305
5	6	2915	515415
3	10	4320	1648640
1	9	8685	7436180
9	2	2850	3312420
9	10	3665	2817125
2	8	13955	5314945
3	8	10040	2228610
8	11	8235	159820

Cost Estimates

63. The individual length and volume estimates given in Table 6 were used to calculate total riprap and granular fill quantities needed for the five alternative configurations shown in Figure 2. Unit prices used in determining costs were obtained from the NAO estimating section and are as follows:

a. Riprap. Riprap stone costs $50.00 per ton and weighs 165 lbs per cu ft in place. Crushed stone for bedding costs $50.00 per ton and weighs 165 lbs per cu ft in place. The geotextile (filter cloth) costs $3.00 per sq yd in place. Forming the dike surface and ditch during riprap placement costs $2.00 per sq yd. Using these figures, the estimated cost of the exterior riprap is $91.62 per sq yd. The estimated cost of interior riprap is $60.12 per sq yd.

b. Dredged Sand Fill. The total cost of dredging, transporting, and placing sand fill, below and/or above water, is estimated to be $1.20 per cu yd in place. Table 7 is the compilation of all quantity and cost calculations for the five alternative configurations.

Table 7

Cost Estimate for the Alternative Configurations

Configuration No.	1	2	3	4	5
Total Dike Length, Ft	29,670	37,475	16,100	14,790	22,595
Total Fill Volume, 1000 Cu Yd	17,699	17,410	11,440	4,326	4,037
Estimated Cost of Fill, 1000 Dollars	21,239	20,892	13,728	5,191	4,844
Total Riprap Area, Sq Yds:					
a. Exterior	89,066	87,732	48,366	44,063	43,112
b. Interior	73,475	92,831	39,840	36,596	55,987
Estimated Riprap Cost, 1000 Dollars	12,578	13,619	7,367	6,238	7,273
Total Cost of Configuration, 1000 Dollars	33,817	34,512	21,095	11,428	12,118
Storage Volume, 1000 Cu Yd*	35,640	45,050	13,440	12,410	20,940
Cost Per Cu. Yd. For Storage	$0.95	$0.77	$1.53	$0.92	$0.58

* Goforth, 1986

PART V: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

64. Based on the studies of the dike system design and of the cost estimates reported herein, it is concluded that:

a. A sand dike extending to el +8 ft MLW may be safely constructed in the proposed expansion areas to the north and west of the existing Craney Island Disposal Area. This is contingent on the actual soil conditions in the marine clay layer being equal to or better than those under the existing facility. It is further contingent on the placement rate of the sand being no greater than that of the original facility.

b. Configuration 5 gives the greatest amount of dredged material storage for the construction dollar. However, the storage volume is intermediate among the five alternatives. Configuration 2, which incorporates Configuration 5, has the next lowest storage cost, and gives the highest storage volume of the five alternatives. Configurations 5 and 2 had estimated costs per cu yd of storage of $0.58 and $0.77, respectively.

Recommendations for Further Work

65. The present study has been made using a number of assumptions regarding soil properties under the proposed dike locations. It has also become apparent that strength increase of the clay under gradually increasing dike load is necessary for dike stability. However, the rate and extent to which this will occur is also unknown. Therefore, it is recommended that additional studies be made:

a. The soil profile along the proposed dike alignments needs to be intensively investigated. Undisturbed tube samples will be needed for laboratory consolidation tests. It is suggested that field vane shear and/or field cone penetrometer investigations be made to extend the sample boring data rather than relying on laboratory strength tests. Of concern is the rather high probability of sample disturbance using tubes in the soft clays.

b. The effective control of construction operations to prevent a stability failure requires that the time rate relationship between load placement and increase in shear strength in the soil profile be thoroughly investigated. Although some information regarding non-unifotm loading can be obtained from the present study (the entire embankment is a non-uniform load), time rate studies should also include evaluation of non-uniformity of loading.

REFERENCES

CAGE, G-CASE Task Group on Slope Stability. 1986. "User's Guide: UTEXAS2 Slope-Stability Package; Volume 1 User's Manual", Instructional Report GL-86-n, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Fowler, J., Edris, E. V., Hanks, W. L., and Holloway, T. K. 1986. Perimeter Dike Stability Analysis, Craney Island, Norfolk District." Technical Report (in preparation), U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Goforth, G. F. 1986. "Disposal Life Evaluation of Alternative Expansion Configurations for Craney Island Disposal Facility." Technical Report (in preparation), U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Heltzel, S. B. 1986. "Norfolk Harbor Long Term Disposal Study, Virginia, Evaluation of Craney Island Enlargement Alternatives." Technical Report (in preparation), U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Norfolk District, U.S. Army Corps of Engineers. 1953. "General Design Memorandum on Craney Island Disposal Area, Norfolk Harbor, Virginia." Unpublished Technical Report, Norfolk, Virginia.

Norfolk District, U.S. Army Corps of Engineers. 1985. "Work Plan, Norfolk Harbor and Channels, Virginia, Long-Term Disposal, Continuation of Planning and Engineering." Unpublished Technical Report, Norfolk, Virginia.

Office, Chief of Engineers. 1987. "Confined Dredged Material Disposal", Engineer Manual EM 1110-2-5027, Department of the Army, Washington, D.C.

Palermo, M. R., Shields, F. D., and Hayes, D. F. 1981. "Development of a Management Plan for Craney Island Disposal Area." Technical Report EL-81-11, Environmental Laboratory, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Skempton, A. W. 1948. "Vane Tests in the Alluvial Plain of the River Forth Near Grangemouth." Geotechnique, Vol. 1, No. 2.

APPENDIX A: TYPICAL INPUT FOR UTEXAS2 SLOPE STABILITY ANALYSIS PROGRAM

```
HEADING - - - -
CRANEY ISLAND EXTENSION, DEPTH = dd FT
CPROFL = xxx PSF, RATEIN = y.y PSF/FT
NONCIRCULAR SURFACE (SLIDING WEDGE)
PROFILE LINES
   1 1 MOIST EMBANKMENT SAND
   -81 0
   -21 4
   -11 8
    11 8
    19 4

    2 2 SATURATED EMBANKMENT SAND
  -El -dd
    -111 -2
    -81 0
    19 4
    109 -2
    +Er -dd

    3 3 CONSOLIDATING DREDGED MATERIAL
    19 4
    500 4

    4 4 SOFT BOTTOM CLAY
    -500 -dd
    500 -dd
    5 5 DENSE BASE SAND
    -500 -90
    500 -90
```

NOTE: This tabulation is not part of the program input.

Depth dd, FT	Left Edge El, FT	Right Edge Er, FT
05	-201	+199
15	-501	+499
25	-801	+799
35	-1101	+1099

```
  6 6 Water
  -500 0
  -81 0
MATERIAL PROPERTY
  1  MOIST EMBANKMENT SAND
     110
     CONVENTIONAL SHEAR STRENGTH
     0 30
     PIEZOMETRIC LINE
     1
  2  SATURATED EMBANKMENT SAND
     125
     CONVENTIONAL SHEAR STRENGTH
     0   30
     PIEZOMETRIC LINE
     1
  3  CONSOLIDATING DREDGED MATERIAL
     90
     LINEAR INCREASE WITH DEPTH
     35 1.8
     PIEZOMETRIC LINE
     1
  4  SOFT BOTTOM CLAY
     100
     LINEAR INCREASE WITH DEPTH
     xxx y.y
     PIEZOMETRIC LINE
     1
  5 DENSE BASE SAND
     125
     CONVENTIONAL SHEAR STRENGTH
     0 30
     PIEZOMETRIC LINE
     1
```

```
    6 WATER
       62.4
       CONVENTIONAL SHEAR STRENGTH
       0 0
       PIEZOMETRIC LINE
       1
PIEZOMETRIC LINE DATA
    1  62.4 Phreatic Line
    -500 0
    -81 0
    19 4
    500 4
ANALYSIS/COMPUTATION
    NONCIRCULAR SEARCH
    -225 0 0
    -200 -dd 0
    -100 -(dd+1) -90
    -50 -(dd+1) -90
    -11 -dd FIX
    0 +8 0
    20 40
COMPUTE
*************
```